

## CHAPTER D-5 EMBANKMENT SLOPE INSTABILITY

This chapter provides some guidance on selection of soil strengths, pore pressures and loading conditions to consider when evaluating slope stability for risk assessments. Assessing the likelihood of slope instability (which may or may not lead to dam or levee breach) requires consideration of the full plausible range of soil behavior and the relative likelihood of different types of behavior which may vary with time and rate of loading.

### D-5.1 Shear Strength Selection

Shear strength is defined by Duncan and Wright (2005) as, “the maximum shear stress that the soil can withstand.” The proper assessment of shear strength for slope stability analyses is a critical aspect of understanding and predicting levee slope stability performance. Discussion of shear strength characterization and how it applies to slope stability is covered in Chapter 2 and Appendix D of USACE EM 1110-2-1902 Slope Stability and is applicable for levee and embankment dam evaluation.

The selection of shear strength for evaluation of levees and dams has to address uncertainty in strength properties assumed for the stability analysis, and the sensitivity of the outcome to variation in the strengths. Large coefficients of variation for data may be due “...in part because these test data many times reflect small-scale variations in soil properties, and in part because the tests themselves may introduce significant measurement error” (Baecher 2003). Measurement error is due to biases related to issues such as sample disturbance, which often (though not always) tends to reduce soil strength, laboratory test procedures, or model bias where conservatism can be introduced in the model used to represent the soil strength (e.g., linear versus curved failure envelope, especially at low stresses). Random measurement error is the scatter in data due to instrument or operator variability, and it is often assumed to distribute equally (Baecher 2003). Some also believe that the wide variation in properties is due to improper interpretation of loading conditions and associated expected soil behavior and is thus not attributable to the soil, but rather the analyst.



When evaluating the slope stability of embankments with probabilistic methods, mean material properties with probability density functions are often used to develop nodal probability estimates for potential failure modes evaluated using event trees.

### **D-5.2 Conceptual Soil Behavior Framework - Critical State Soil Mechanics**

The critical state soil mechanics (CSSM) concept is based on the observation that when soils are continuously distorted they trend towards and eventually reach a “critical state” where they flow as a frictional fluid with constant effective mean stress, constant deviator stress (i.e., shear strength) and constant void ratio (Schofield and Wroth, 1968 and Shewbridge and Schaefer, 2013). Under this conceptual framework, soils loaded under shear reach a critical density/void ratio where there is no further change in shear stress and no further change in volume. For example, loose (“wet” of critical state) sands tend to contract during shear as grains reorient into a denser pattern until a critical void ratio is reached. Dense sands (“dry” of critical) dilate during shear and eventually reach a critical void ratio or critical state. Fine-grained materials exhibit the same behavior pattern: normally to lightly over consolidated soils (overconsolidation ratio (OCR) up to 2 to 4, “wet” of critical state) tend to contract when sheared in a drained test, whereas highly over-consolidated soils (OCR greater than 2 to 4, “dry” of critical state) tend to dilate. This provides an understanding of how and when positive or negative pore-water pressures may develop during undrained shear.

Loose and normally- to slightly-over-consolidated soils (i.e.,  $OCR < 2-4$ ) will develop positive pore-water pressures during undrained shear and are modeled using undrained shear strengths. Dense, highly over-consolidated soils will develop negative pore-water pressures during undrained shear, but over time, these negative pore pressures will dissipate and their beneficial effect on strength and stability will be lost for long-term stability. When the OCR is between 2 and 4, often the differences in drained and undrained strengths are small and drained or undrained shear strengths may be used, recognizing that the effects on slope stability results are perhaps even less than the uncertainty associated with material types, investigation, and testing.

Consolidation tests or other similar test results are necessary to estimate the pre-consolidation pressure and the OCR for the foundation and embankment soil profile in effect during the load condition being evaluated. Consolidation tests, therefore, are useful to understand soil behavior during shear. Compaction of embankment materials typically results in a heavily over-consolidated state for shallow depths that when subjected to shear results in dilation. This is a positive attribute since dilation can initially create negative pore-water pressure and increase effective stress and strength. However, at depth, the weight of the embankment may be sufficient to bring the OCR below 2 to 4; thus compacted or geologically overconsolidated materials may become normally consolidated, so they would generate positive pore pressures during shear (e.g., Morgenstern et al, 2015).

Drained Versus Undrained Loading Times. As discussed above, during shear, depending on the stress conditions and void ratio, soils have the potential to develop positive or negative shear-generated “excess” pore pressures. If loading is faster than those excess pressures can dissipate, undrained strengths will usually control short-term stability. From classic consolidation theory, Duncan and Wright (2005) suggest a practical measure to assess whether the loading will be faster or slower than drainage can occur:

$$t_{99} = 4(D^2/c_v)$$

**Equation D-5-1**

where  $t_{99}$  is the time required for 99% of the excess shear- and consolidation-induced pore pressures to dissipate,  $D$  is the shortest distance that the water must travel to flow out of the soil mass (i.e., the drainage distance from consolidation theory, typically either  $1\times$  or  $1/2\times$  the thickness of the soil layer in question, depending on drainage conditions at the boundaries), and  $c_v$  is the coefficient of consolidation, which reflects the effects of soil permeability and compressibility, affecting the rate of consolidation. If the loading application rate is faster than this somewhat conservative measure of the length of time that substantial excess pore pressures will remain different from the long-term equilibrium pore pressure levels, then undrained conditions should be considered for analysis of stability at the time of maximum load application in the short-term. If the loading duration is approaching or longer than this often, though not

always, conservative measure of the time for significant excess pore pressures to dissipate, then drained strength conditions should be considered for analysis of stability for the long-term condition.

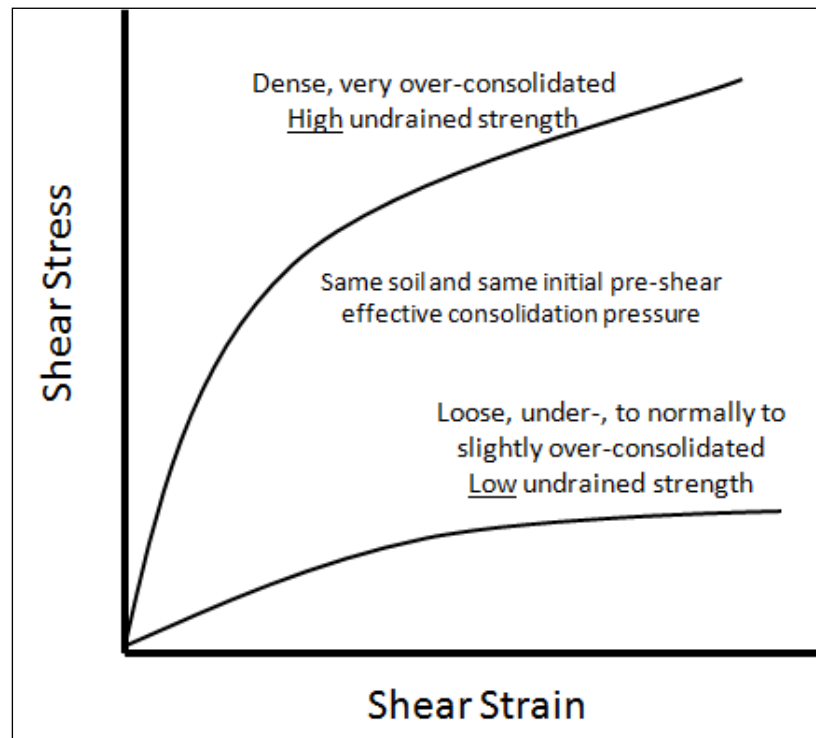
From Critical State theory, it is recognized that for materials that generate positive pore pressures when sheared (i.e., loose, or normally to slightly over-consolidated fine-grained soils), then undrained strengths are likely to be controlling. For materials that generate negative pore pressures when sheared (i.e., dense or highly over-consolidated fine grained soils), then drained strengths are likely to be controlling, long-term. If the loading is applied slowly enough for much, but perhaps not all of the shear induced pore pressures to dissipate, then more sophisticated soil strength analyses, such as anisotropic consolidated strength assessments, may be appropriate. The following paragraphs describe how drainage conditions affect the strength of loose and/or under, normally- to slightly over-consolidated soils and dense and/or highly over-consolidated soils.

### **D-5.3 Undrained Strengths**

Undrained shear strengths are typically assigned to fine-grained soils that are loaded faster than excess pore pressures generated by consolidation and shear can dissipate. Such conditions typically occur during or at the end of construction, where little time has passed, so no consolidation or corresponding decrease in void ratio occurs that would produce an increase in shear strength in saturated soils. In this case, the shear loading comes from the placement of the embankment fill. Relatively rapid loading can also occur during flooding and reservoir rise. In these instances the embankment may or may not have time to consolidate under its own weight and other forces acting on it; the undrained shear strength must reflect the degree of consolidation. Finally, undrained strengths may also control during very rapid loading of even relatively coarse free-draining soils, such as during seismic shaking, when pore pressure generation can lead to liquefaction.

If the soil is loose and/or under-, normally, or only slightly over-consolidated, the undrained strength will be very low relative to denser material or to the same soil under drained conditions

(Figure D-5-1). In contrast, if the soil is dense and/or highly over-consolidated, the undrained strengths will be very high. For dense/over-consolidated materials, often high short-term undrained strengths will not control but rather the lower drained strengths will control. Conceptually, undrained conditions can be modeled using effective stress parameters, but pore water pressures generated during shear must be estimated and are often too difficult to reliably model (VandenBerge et al, 2015). Instead undrained strengths, estimated based on effective-stress conditions prior to the applied load condition, are routinely used for fine-grained soils in end-of-construction and other short-term loading cases such as may occur during flood loading (i.e., it is assumed that no consolidation occurs due to the additional loading whether from construction, flooding, or seismic loads).



**Figure D-5-1. Typical Undrained Stress Strain Curves for Loose and Dense Soils.**

#### **D-5.4 Drained Strengths**

Under static loading, most soil materials that consist of clean, higher permeability sands and gravels can be assumed to drain during compression and shear, and their shear strength is a

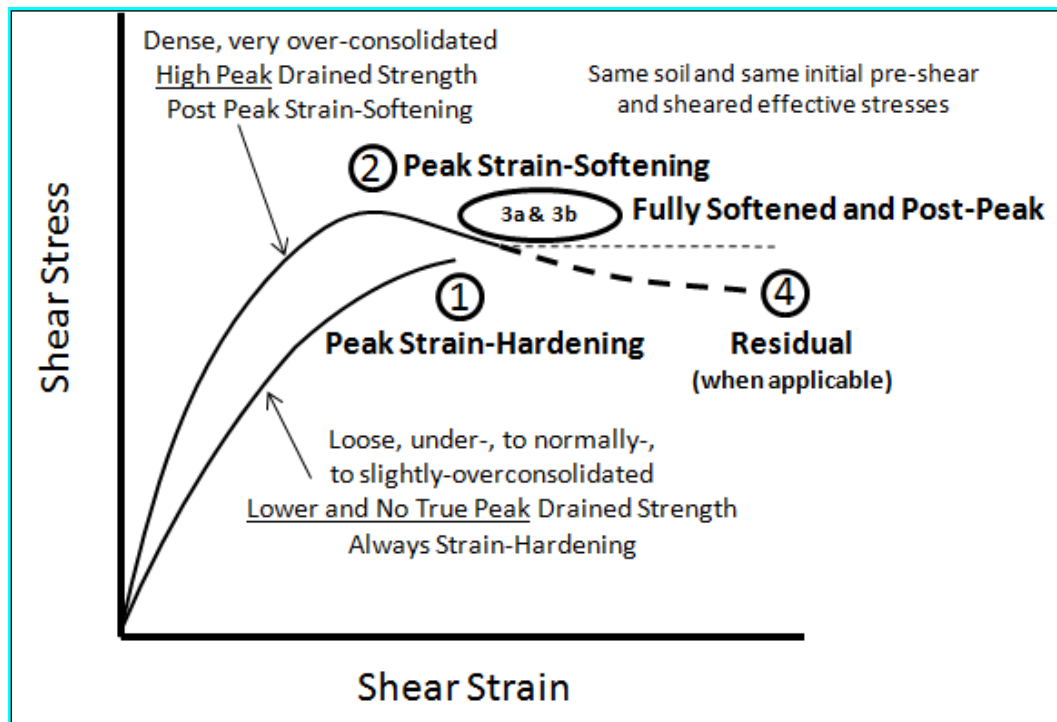
function of effective confining stress. During most typical loadings, including a fast-rising river or hurricane storm surge, these higher-permeability materials are expected to drain, so any shear-induced excess pore water pressures will dissipate nearly instantaneously. Excess pore pressure in clays may also drain if loading is slow enough that excess pore-water pressures generated by compression or shear dissipate, leading eventually to a non-transient (steady state) seepage condition. Between sands and clays can be a myriad of soil types with drainage conditions dependent on loading rates, as discussed below.

Drained strength is generally expressed using effective-stress parameters, so an estimate of expected pore-water pressures is required. The pressures can be represented in stability analysis by one or several piezometric surfaces or by a field of porewater pressures. These would be determined from seepage analysis by flow net or finite-element seepage analysis supplemented and verified by any piezometric measurements and seepage observations that are available. Depending on the case being analyzed, these may be either steady-state seepage or predicted transient seepage resulting from reservoir fluctuation or other cause. The following subsections describe the types of behavior and strengths that should be considered for drained strength estimation.

#### **D-5.5 Strain Hardening, Strain Softening, Peak, Fully-Softened, Post-Peak, and Residual Drained Shear Strengths**

The methods for selecting and assigning drained shear strength properties to levee embankment and foundation materials range from estimating strengths using empirical relationships (related to simple index testing) to comprehensive in-situ and detailed laboratory shear strength testing combined with careful evaluations of the full range of soil behavior over the range of potentially induced strains. Published relationships may be and are often used for preliminary analyses, but advance risk analysis projects may warrant site-specific testing. From site-specific testing a decision is made on the appropriate failure criterion, followed by selecting the associated shear stress. The selected shear stress values are then used to establish the shear strength envelope or simply “shear strength” to be used in modeling. There are conditions or effects, such as creep and progressive failure, where the peak shear stress from testing may overestimate the shear

stress concurrently mobilized along an entire potential slip surface at time of failure. Several options for defining failure are discussed below in the numbered paragraphs correlating with the conditions of soil strength and deformation in Figure D-5-2.



**Figure D-5-2. Generalized Drained Stress Strain Curves for Loose and Dense Soils and Types of Strengths Considered for Use in Stability Analyses.**

#### **D-5.5.1 Peak Shear Strength for Relatively Loose Strain-hardening Soils**

For materials that exhibit strain hardening with no post peak softening (for example, loose to medium dense sands and under-, normally-, and slightly over-consolidated fine grained soils), peak shear strengths, defined by maximum stress difference or maximum stress ratio, are generally controlling and are recommended for use in stability analyses. See “1” on Figure D-5-2.

#### **D-5.5.2 Peak Shear Strength for Relatively Dense Strain-softening Soils**

For materials that do exhibit strain softening effects and that will not be loaded to shear stresses greater than peak, peak shear strength can be considered in slope stability analysis. Such materials include dense sands that are denser than their critical state, some well compacted

and/or desiccated lean clays (i.e., soils with little to no volume changes during climate and moisture changes), and materials not subject to creep and progressive failure. These materials typically experience little change in void ratio with age and the peak shear strength is less likely to degrade significantly with time. See “2” on Figure D-5-2. Analysts are cautioned though that if peak strengths are exceeded in any part of the slope, strain softening behavior can result in strain incompatibility where peak strengths may be achieved and surpassed along part of the slip surface and not attained or surpassed along other parts, such as occurs during progressive slope failure. Where strain softening or progressive failure can occur, fully softened, post peak and residual strengths, described in the following paragraphs may be more appropriate for use in computations.

#### **D-5.5.3 Fully Softened Shear Strength for Strain-softening Soils**

For materials that do exhibit strain softening effects and that may be loaded to shear stresses greater than peak shear strength, or greater than the age reduced peak shear strength, including slopes vulnerable to progressive failure, the analyst should consider whether fully softened shear strength (FSS) should be used in slope stability analysis. FSS is a concept introduced by Sir Alec Skempton (1970) and is complimentary to the Critical State soil behavior model described above. As reported by Duncan et al. (2011):

Skempton suggested that “we may say that the fully softened strength parameters  $c'$  and  $\phi'$  are equal numerically to the peak strength parameters of the normally consolidated clay.” Equating the [drained] strength of normally consolidated test specimens to the “fully softened” strength in this manner is a somewhat conservative approximation. This observation should be viewed as an empirical conclusion rather than a fundamental principle of soil behavior.

FSS is typically used to represent long term drained shear strength conditions of stiff fissured clays and shales and compacted fat clays and are represented with drained material properties modeled in terms of effective stress. It is typically used to assess the factor of safety for “first time” slides in soil and soft rock bodies that have not yet developed distinct large strain failure surfaces. See “3a” on Figure D-5-2.



An FSS testing standard is available for the ring shear device in ASTM D 7608 and a suggested test method for the direct shear device can be found in Stephens (2014). The triaxial apparatus can sometimes also be used to assess fully softened shear strengths, but no widely-accepted standards are available at this time. The concept of FSS has been applied and is generally accepted as an analysis approach for assessing long term performance of cut slopes, post weathering and aging, and has also been applied to levee slope evaluation, particularly for fat clays subject to significant desiccation and flood rewetting, often experienced in arid environments. Weathering processes of wetting and drying and freezing and thawing result in volume changes and micro-straining of expansive materials. Levees constructed of expansive fat clays are subject to greater cyclic volume changes and may form cracks and fissures with associated accelerated aging and environmental changes. In this way the peak shear strength due to over consolidation or compaction-induced “quasi-over consolidation” in levees, is reduced to a lesser value. Surface sloughs, and in some cases relatively deep slides (compared to the height of the levee), can result following extended dry periods followed by significant rainfall. Desiccation cracking can lead to more rapid saturation of an embankment or slope than non-desiccated conditions and further contribute to slope instability. Typically, maintenance-type slides can be repaired prior to flooding, but prolonged extensive rainfall and subsequent flooding are not mutually exclusive events and any slide that occurs during flooding can be problematic. Compounding the loss of strength from weathering, levees that experience desiccation cracking are subject to leakage through transverse cracks during flooding as the cracks may or may not fully heal. Open desiccation cracks were experienced during the 1997 flood of record in Grand Forks, North Dakota, and leakage through the cracks had to be addressed during flood fighting (Schwanz, 2015). Often, more representative slope stability analysis results can be achieved using a curved failure envelope to describe the high sensitivity of FSS strengths to mean stress levels, helping to avoid trivial and often too low shallow slide factors of safety.

#### **D-5.5.4 Post-peak or Ultimate Shear Strength for Strain-softening Soils**

As discussed above, the FSS concept provides a means of assessing and assigning long term shear strengths to over consolidated soils, such as high plasticity clays subject to weathering, creep and progressive type failure. Unfortunately, FSS testing is not routine in many laboratories

and further, without established standards for performing FSS testing using conventional triaxial or direct shear apparatuses, analysts may not have a readily available method to assess FSS strengths

In some situations, time and cost constraints have led to using other more common standard tests to estimate FSS using measured post peak shear strengths. Post peak shear stress, as the name implies, represents the shear stress measured at strain or displacement increments during continued loading after the maximum shear stress in the specimen has developed. Because there is not a stress state associated with a physical condition, it is difficult to choose, with consistency, a single value for analysis strengths. To approximately estimate strengths and to address concerns of progressive failure and strain compatibility along potential slip surfaces, as well as the propensity for creep, the selection of shear stress associated with strain past peak has been used when performance has been verified with local practice. See “3b” on Figure D-5-2.

#### **D-5.5.5      Residual Shear Strengths**

Residual shear strength is used to characterize materials that have already undergone failure and large strains along distinct sliding surfaces. Such a condition may occur where an embankment is constructed near or on an existing slide in a clay foundation, or when excavation unloads the toe of a slide. Particularly for clayey material, with plate-shaped particles, the large strains can cause reorientation of particles so they slide over each other more easily, without dilation or contraction. (See curve "4" on Figure D-5-2.) In plastic clays, the residual friction angle can be a small fraction of the peak friction angle. Residual strengths should be assigned to failure surfaces in slopes with previous slides or other geologic features that have experience large strains in the past, leading to the development of a distinct failure surface. One of the best methods to assess residual strengths is back analysis of failed slopes. Back analysis requires good estimates of the pore pressures that existed at the time of the slide, and accounting for any significant 3D effects. The results of back analysis can be used in conjunction with laboratory shear testing, most commonly reversal direct shear tests. If the position of a preexisting failure surface is known (from inclinometers or field observation), then this position should be used in the back analysis or as an alternative, a thin zone of material at the residual shear strength can be

used following the slide location. (This does not mean that other sliding surfaces can be ignored.) If the position of the failure surface is not known, a search procedure should be used in both forward and back analysis. Residual shear strength is typically expressed as a drained friction angle in effective stress analysis.

#### **D-5.6 Pore Water Pressures and Associated Strengths for Analysis**

As discussed above, strengths used in slope stability analyses are often described in terms of total stress parameters for undrained shear strengths and effective stress parameters for drained shear strengths; some analysis conditions may require a mix of undrained total and drained effective stress strength models, depending on the material types, configurations and densities of the embankment and foundation materials, and the speed and duration of the loading condition. Pore pressure (water or air) is implicitly addressed in the testing and selection of undrained shear strength properties used in total stress analyses, but must be explicitly expressed for drained effective stress strength assessments. While conceptually valid, matric suction and pore-air pressures are typically ignored (compressibility of air is assumed not to affect inter-granular stress and pore-moisture suction is difficult to reliably predict) when assessing drained effective stresses for design, but may be appropriate for risk analysis purposes. In contrast, positive pore-water pressures, or changes in pore-water pressures, must almost always be accounted for. Positive pore-water pressures can originate from several sources such as:

- flood loading;
- naturally existing groundwater or long term seepage conditions;
- fill placement or loading;
- excavations;
- poor surface water control;
- rainfall infiltration;
- broken water lines;
- and other loading conditions that change horizontal and/or vertical total stresses.

Negative pore-water pressures that are associated with matric suction from capillary action are sometimes considered in forensic and risk evaluations. When the concepts of unsaturated soil mechanics are used in other situations, its use must be clearly described using the latest principles (that is, use of  $\phi^b$ ) (GEO-SLOPE International 2007).

Computer analysis tools to perform transient seepage analyses are increasingly available, but the ability to measure or otherwise evaluate parameters needed for unsaturated soil mechanics is not sufficiently established for design use and is still in question for risk analysis (VandenBerge et al 2015). Further, at this time, there are no readily available programs that can easily evaluate dissipation of shear-induced positive or negative pore pressures. Finally, transient seepage analyses tools do not yet have a proven track record, with calibrated model results compared to field performance case histories. As such, transient results are not yet considered robust and reliable for routine design and may not be appropriate for risk analyses. Nevertheless, they can be effectively used to evaluate sensitivity to the parameters affecting saturation and development of pore water pressures, and to help guide the analysts to a better-informed opinion about factors affecting performance and strength.

#### **D-5.6.1 Total Stress Methods to Evaluate Soil Strength**

When using total stress methods to evaluate undrained soil shear strengths (i.e., the undrained strength is set equal to the undrained cohesion ( $c = s_u$ ) and the undrained friction angle  $\phi_u = 0$ ), changes in the total stress do not affect shear strength. Therefore, pore-water pressure is often denoted as having a value of zero or it is explicitly not used to assess soil total stresses, depending on the input recommendations of the analysis software package used. So, even though a piezometric condition may be explicitly specified, in most stability analysis programs it will have no effect on the estimated undrained strength of the soils modeled using total stress parameters

Typically, undrained strengths for under-, normally and slightly overconsolidated fine grained soils are evaluated by first assessing the pre-loading (e.g., pre-flood) effective stress profile, which is then used to estimate undrained strengths available during loading using appropriate

undrained strength models, such as SHANSEP (Ladd and Foott, 1974 and Ladd and DeGroot, 2004) or  $S_u/p'$  (Duncan and Wright 2005), accounting for various factors, such as anisotropic consolidation, aging, etc..., when appropriate.

Sometimes, soils that are partly saturated may develop increased shear resistance as a function of an increase in total stress (i.e., total stress cohesion  $> 0$  and total stress  $\phi > 0$ ) and may also be modeled using total stress parameters. Again, zero pore pressures are still often explicitly specified in the stability analyses to avoid computational errors. More often, use of simple drained friction angles and curved failure envelopes yields more reliable and representative results, avoiding complications with some of the above approximate methods (see USACE EM 1110-2-1902 Slope Stability 2003, for more information).

#### **D-5.6.2 Effective Stress Methods to Evaluate Soil Strength**

Pore-water pressures need to be defined when using effective stress methods to evaluate drained soil shear strengths. Common options for defining pore-water pressures include the following:

- Using the phreatic surface (water table) as a piezometric surface and computing the pore-water pressure as the vertical distance from the piezometric line to the point of interest, multiplied by the unit weight of water. Strictly speaking, a single piezometric line is only correct if there is no vertical seepage gradient, although it is often a reasonable approximation.
- Specifying a set of piezometric lines in an aquifer from piezometer data and/or a closed form solution (such as that is, blanket theory often used in levee evaluations - USACE 1956).
- Specifying pore-water pressure based on a graphical flow net by identifying pore-water pressures at points and interpolating between those points as needed.
- Importing pore-water pressures from finite element/difference solutions.
- Specifying pore-water pressures based on field instrumentation by identifying pore-water pressures at points and interpolating between those points as needed.

- Applying a pore-water pressure coefficient  $r_u$  (an older generally outdated method, rarely used, but may occasionally be employed for evaluating clay embankments using FSS where  $r_u \leq 0.6$ ).

Using a phreatic surface or a single piezometric line to define pore-water pressures in slope stability models has been used successfully on many projects and remains a reasonable approach but can lead to unconservative results in the form of higher computed factors of safety when there is upward flow of water near the embankment toe (Duncan et al. 2005; Perri et al. 2012). The reverse can occur when a blanket drain or pervious foundation causes downward flow. As several geotechnical software suites offer FEM seepage analysis tools that facilitate automatic import of pore pressures into limit-equilibrium slope stability analyses or coupled with stress deformation software, it is now relatively easy to perform stability analysis using robust steady state seepage pore-water pressure regimes and is preferred.

Steady-state seepage conditions are assumed most commonly; however, transient seepage may also need to be considered. When transient conditions are being considered, analysis assumptions, such as whether unsaturated and undrained conditions will prevail on projects before unsaturated conditions are relied upon for embankment stability, will receive high scrutiny. Often, the cost to explore and identify all potential defects in the embankment and foundation that could often lead to significant violations of the assumptions will be cost prohibitive. An unidentified defect in presumed unsaturated clay, such as a sand lens or layer, variable clay properties and degrees of saturation, desiccation cracks, an abandoned utility pipe or an animal burrow, may significantly reduce the time for saturation to occur. In cases where unsaturated conditions are being relied upon, monitoring of embankment performance is likely even more important than for embankments designed for long-term steady state seepage conditions. In addition, worst reasonable case large, long-duration floods in flashy watersheds (ones where discharges increase and decrease rapidly in response to precipitation) may be very infrequent, so there would be little opportunity to observe behavior with elevated reservoir levels to confirm that performance is consistent with analysis expectations, and to respond before there are negative consequences. If transient conditions are required for acceptable system

performance and short-term unsaturated or high undrained strengths are used in the short-term stability analyses, significant contingency plans to identify and rapidly respond to unexpected problems may be required, particularly for high consequence systems, such as large high-hazard dams and urban levees (Shewbridge and Schaefer 2013).

#### **D-5.7 Loading Conditions**

Throughout the life of a project there is always at least one short-term and one long-term loading condition and there may be other interim conditions that govern slope stability. The various loading conditions to which an embankment and its foundation may be subjected and which should be considered in analyses are designated as follows: Case I, end of construction; Case II, sudden drawdown from full flood stage; Case III, flood loading; and Case IV, seismic (Figure D-5-3). Each case is discussed briefly in the following paragraphs and the applicable type of shear strength is suggested. With the exception of undrained soil response during flood loading, more detailed information on applicable shear strengths, methods of analysis, and assumptions made for each case are presented in USACE EM 1110-2-1902 Slope Stability (2003). The undrained soil response during flood loading is incorporated in this guidance to address the development of positive pore-water pressure generation during shear and the use of undrained shear strengths for fine grained soils in those situations, which is not adequately addressed in EM 1110-2-1902.

Case I - End of Construction. This case represents undrained conditions for low-permeability embankment and/or foundation soils, where excess pore water pressure is present because the soil has not had time to drain since being loaded in compression and shear. For low-permeability materials that would be loaded in an undrained manner, results from laboratory unconsolidated-undrained tests (UU) and vane shear (VST) strengths are applicable to fine-grained soils loaded under this condition. CPT correlations to undrained shear strength can be used if correlated to UU or VST results. Low permeability materials are often represented in stability analysis programs using undrained strengths estimated using pre-loading effective stresses, such as with the SHANSEP (Ladd and Foote 1974) or  $s_u/p'$  methods (Duncan and Wright 2005) and when good quality oedometer testing is available, but correlations alone are not a complete replacement for laboratory or field strength testing. For relatively high-permeability materials

that will be loaded in a drained manner, results of consolidated-drained (CD) tests can be used to represent their effective stress strengths. The end of construction condition is applicable to both the riverside and landside slopes of levees and the upstream and downstream slopes of dams.

Case II - Sudden Drawdown. This case represents the condition whereby a prolonged flood stage or even normal storage saturates much of the upstream portion of the embankment, and then flood load or the reservoir falls faster than the soil can drain. This can cause both higher pore pressure directly, and by causing excess pore water pressure to develop from undrained shear. This can result in the upstream dam or waterside levee slope becoming unstable. For the selection of the shear strengths, see Table 8-1 and EM 1110-2-1902 for more information. Often the biggest challenge in this loading condition is evaluating appropriate water surface elevation reductions for the drawdown analysis, which may be affected by normal hydrologic and hydraulic conditions as well as abnormal level changes due to unexpected events. Rarely does this loading condition lead to embankment breach, but it may need to be considered in the risk assessment.

Case III - Flood Loading. Flood loading applies when river stages exceed the landside levee toe elevation or the reservoir stage rises above normal storage behind the dam. This load case can include the steady seepage condition when pore-water pressures from seepage fully develop, and pore-water pressures due to shear dissipate (for example, sand levees on sand foundations). But it also addresses the likelihood that embankment and foundation materials may only partly saturate and/or shear-induced positive pore water pressures will not dissipate during flood loading such as for projects with fine grained soils that prior to flood loading are normally to lightly over-consolidated with an OCR less than say 2-4 (note that in contrast to this, for soils that are heavily overconsolidated, the negative pore water pressures that generate during shear due to dilation are assumed to dissipate). This load case occurs when the flood loading remains at or near the selected flood stage and a condition of steady seepage may or may not develop in all of the low-permeability soils. As combined flood and shear induced pore-water pressures are difficult to estimate, the strength of the fine-grained undrained soils are represented using undrained strength parameters and the undrained shear strengths are estimated based on pre-flood effective



stresses. Soils with an  $OCR < 2$  typically exhibit undrained shear strengths that are lower than the drained strengths associated with long-term steady seepage; therefore the undrained strength determined from pre-flood effective stresses should be used. Likewise, for soils with an  $OCR > 4$  it is typical that the drained strength associated with long term steady seepage conditions will be lower than the undrained strength, so the drained strength should be used for design. Often, depositional conditions result in stratigraphy also containing coarse-grained, free-draining soils and these strata may respond quickly to river stages with pore-water pressures corresponding to steady state seepage conditions and fully dissipated shear induced pore water pressures. The free-draining soils are thus characterized using effective stresses estimated using the steady state seepage pore pressures and associated drained effective stress shear strength properties (see Appendix I). This loading condition may be critical for landside slope stability. Again, it is important to recognize that the weight of the embankment may be sufficient that compacted materials near the base and overconsolidated foundation materials would be loaded to new consolidation pressures greater than past maximums. These materials could then behave as normally consolidated soils that would generate positive excess pore pressures during shear (e.g., Morgenstern et al, 2015).

Case IV – Seismic. Embankments constructed of loose cohesionless materials or founded on loose cohesionless materials are particularly susceptible to failure due to liquefaction during earthquakes that may occur any time and are evaluated using expected groundwater conditions. More details on seismic analysis are covered in the chapter on Seismic Risks for Embankments.

**Table D-5-1**  
**Summary of Typical Analysis Conditions**

Analysis Condition	Shear Strength <sup>a</sup>	Pore Water Pressure
Case I. During Construction and End-of-Construction	<p>Free draining soils - use drained strengths</p> <p>Low permeability soils wet of critical – use <u>undrained</u> strengths based on pre-construction effective stress conditions for soils with <math>OCR &lt; 2</math> to 4</p> <p>Low permeability soils dry of critical – use <u>drained</u> strengths when <math>OCR &gt; 2</math> to 4.<sup>b</sup></p>	<p>Free draining soils - Pore water pressures can be estimated using analytical techniques such as hydrostatic pressure computations for no flow or steady seepage analysis techniques (flow nets, finite element/difference analyses).</p> <p>Low permeability soils wet of critical – use total stresses with pore water pressures set to zero in the slope stability computations for materials with <math>OCR &lt; 2</math> to 4</p> <p>Low permeability soils dry of critical – use effective stresses with appropriate construction pore pressures, often assumed to be hydrostatic <math>OCR &gt; 2</math> to 4</p>
Case II. Sudden Drawdown Conditions	Free draining soils - use drained strengths	Free draining soils - First stage computations (before drawdown) - steady state seepage pore pressures as described for steady state seepage condition. Second and third stage computations (after drawdown) - pore water pressures estimated using same techniques as for steady seepage, except with lowered water levels.

	<p>Low permeability soils - Three stage computations: First stage use effective stresses; second stage use undrained shear strengths and total stresses; third stage use drained strengths (effective stresses) or undrained strengths (total stresses) depending on which strength is lower - this will vary along the assumed shear surface.</p>	<p>Low permeability soils - First stage computations – steady state seepage pore pressures as described for steady state seepage condition.</p> <p>Second stage computations - Total stresses are used, pore water pressures are set to zero.</p> <p>Third stage computations - Use same pore pressures as free draining soils if drained strengths are being used; where undrained total stress strengths are used, pore water pressures have no effect and can be set to zero.</p>
Case III. Flood Loading	<p>Free draining soils - use drained strengths. Residual strengths should be used where previous shear deformation or sliding has occurred.</p> <p>Low permeability soils wet of critical – use <u>undrained</u> strengths based on pre-flood effective stress conditions for soils with <math>OCR &lt; 2</math> to 4</p> <p>Low permeability soils dry of critical – use <u>drained</u> strengths using steady state seepage pore pressures.<sup>b</sup></p>	<p>Free draining soils - Pore water pressures can be estimated using analytical techniques such as hydrostatic pressure computations for no flow or steady seepage analysis techniques (flow nets, finite element/difference analyses).</p> <p>Low permeability soils wet of critical – use total stresses with pore water pressures set to zero in the slope stability computations for materials with <math>OCR &lt; 2</math> to 4.</p> <p>Low permeability soils dry of critical – use effective stresses under steady state seepage flood loading for materials with <math>OCR &gt; 2</math> to 4.</p>

Analysis Condition	Shear Strength <sup>a</sup>	Pore Water Pressure
Case IV. Seismic	(see Chapter on Seismic Risks for Embankments for more details)	

<sup>a</sup> Effective stress parameters can be obtained from consolidated-drained (CD) tests (either direct shear or triaxial) or consolidated-undrained (CU) triaxial tests on saturated specimens with pore water pressure measurements. Direct shear or Bromhead ring shear tests should be used to measure residual strengths. Undrained strengths can be obtained from unconsolidated-undrained (UU) and direct simple shear tests. Undrained shear strengths can also be estimated using consolidated-undrained (CU) tests on specimens consolidated to appropriate stress conditions representative of field conditions, but these strengths may be unconservative. The CU or “total stress” envelope, with associated  $c$  and  $\phi$  parameters, should not be used for Load Cases I and III, but instead use undrained shear strength increase with depth or as appropriate. OCR is estimated based on the maximum past pressure and the effective stress prior to construction (Case I) or flood load (Case III) as this stress state, and corresponding undrained shear strength, exists prior to loading. For sudden drawdown (Case II) refer to EM 1110-2-1902 for selection of undrained shear strengths from CU tests.

<sup>b</sup> For saturated soils with  $OCR < 2$  to 4, use  $\phi = 0$ .

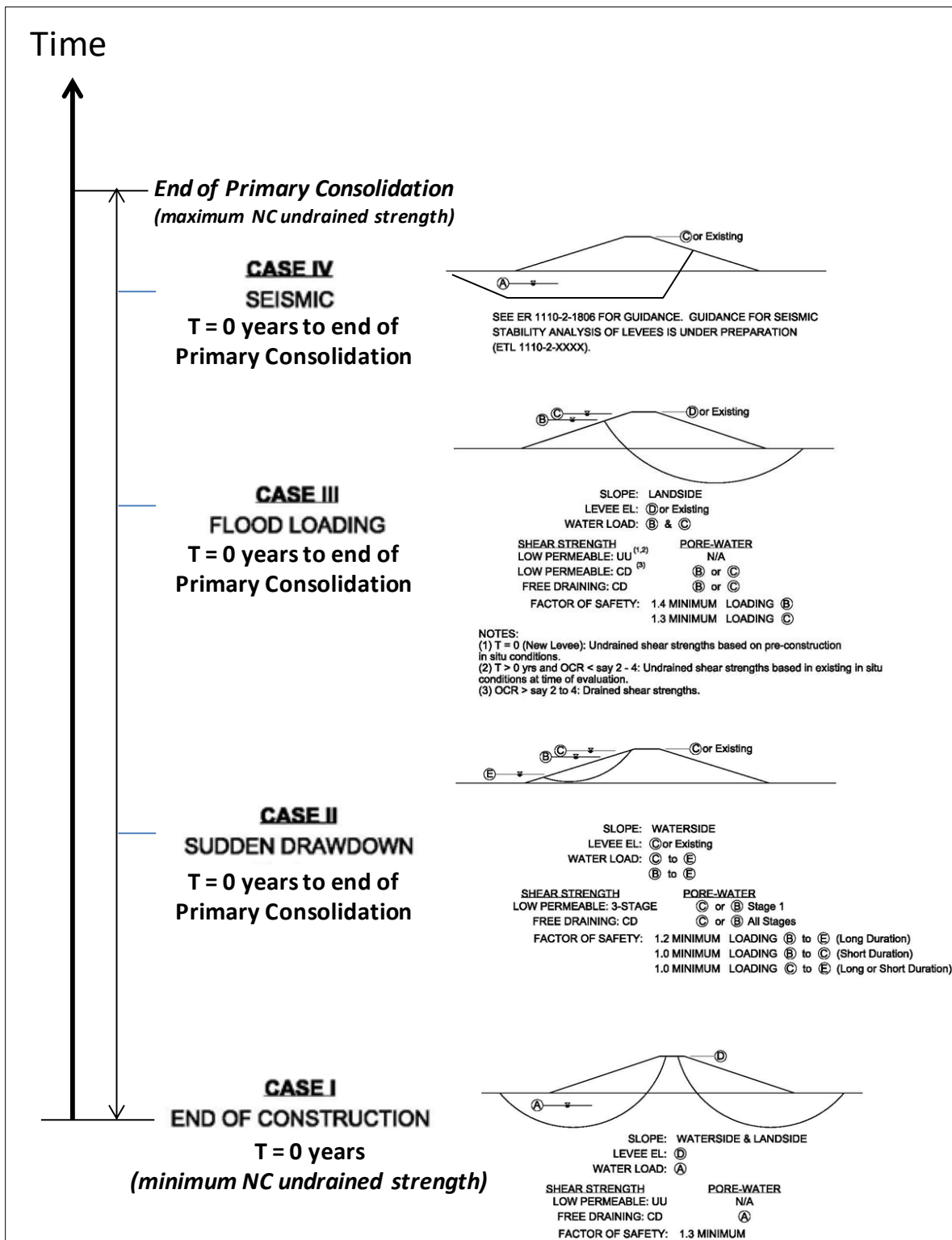


Figure D-5-3. Example Slope Stability Loading Conditions and Typical Associated Design Factors of Safety for Levee Evaluations. Similar Conditions Apply to Dams.

## **D-5.8 References**

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